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PILE FOUNDATIONS FOR BUILDINGS

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SOIL MECHANICS AND FOUNDATIONS DIVISION

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PILE FOUNDATIONS FOR BUILDINGS

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SYNOPSIS

There is derived herein a method of determining the capacities, lengths and spacing of foundation piles which are to be driven and remain in contact with the soil and are to be loaded axially. A design procedure for application of the method and a field procedure to carry out the intent of the design are presented.

INTRODUCTION

A structural engineer faced with the necessity of designing a pile foundation and estimating the lengths of piles required is confronted with a wealth of vague information. Out of it he must develop a definite method of dealing with his problem.

Well acquainted with the methods of engineering mechanics, and the comparatively definite properties of the materials used in the superstructure, he is apt to be dissatisfied with the tools he finds available in connection with pile foundations. At this point; he may either work out for himself a method that seems to have some rational basis; he may give up and swallow a dynamic formula whole; or he may load test a pile in the hope of being able to extrapolate the results to produce a rational design of a whole foundation. Local experience, properly interpreted, may keep him from going too far astray. Local experience, however, may not cover his needs.

Such situations have led to the development of the method here presented. It has, in its various stages of development, been used on the following projects:

1. Washington, D.C., U.S. Court House for the District of Columbia. (2639 piles).
2. Gravelly Point, Va., National Airport, South Extension to Terminal Building. (461 piles).
3. Bethesda, Md., National Institutes of Health, Shops, Laundry and Storage Building (2018 piles); and Boiler House (1337 piles).

In every case but one the total length of piles as driven has been very close to the estimated total length. There was an average underrun of 2.87 feet per pile on the Court House, 5.53 feet per pile on the Terminal Building, and 1.62 feet per pile on the Boiler House. There was an average overrun of 0.98 feet per pile on the Shops, Laundry and Storage Building.

The excessive underrun on the Terminal Building led to a modification of the procedure for non-cohesive soils to take account of shape of grain and grading.

Certain related problems that are covered in detail elsewhere will not be treated here. Such problems include the proportioning of a pile to carry its

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load as a compression member, and the problem of differential settlement which is not essentially different for pile foundations than it is for spread footings.

The design of a pile foundation shares a special feature with all foundation problems. The qualities of the soil, one of the materials involved, must be assumed from tests on very small and widely distributed samples. These qualities differ between the various samples enormously more than the qualities of other building materials such as steel and concrete.

The literature of pile foundations makes it plain that the use of dynamic formulas for the determination of pile length is valid only for single piles in a very limited range of soils. The results of load tests are valid only for individual piles or for groups of the size tested. There is a need for more specific design and construction procedures. Even where load tests are to be made, some method of determining the proportions of the pile or group to be tested is needed.

Theory

A successful pile foundation requires that the following four conditions be fulfilled.

1. Each pile must be capable of carrying its load as a compression member.
2. Each pile must be driven far enough into the soil to transmit its load to the soil by bearing on its point and shear on its periphery without excessive motion relative to the soil.
3. The soil at the point of each pile must be capable of carrying the full pile load on the area available for that purpose with a suitable factor of safety against failure. For a single pile, the area available for support will vary with the length of embedment of the pile in the supporting soil and with the quality of the soil. For an interior pile in a large group, the available supporting area cannot be much larger than the area bounded by a line midway between piles. For piles in small groups, say four or less, an intermediate value will obtain.
4. The soil at any distance below the pile points must be capable of carrying the full pile load on the larger area available because of the lateral spread of the pile loads, again with a suitable factor of safety against failure.

In the development of rules for establishing pile sizes, lengths and spacings that will satisfy the foregoing conditions, the following simplifying assumptions are made.

1. A pile receives no support from the unsatisfactory strata through which it is driven to reach the bearing strata. It delivers its full load, therefore, to the bearing strata by shear on its periphery and bearing on its point in those strata.
2. The properties of the supporting strata are those of the weakest included stratum.
3. The unit pressure on the pile point is the same as the vertical unit pressure alongside the pile, at the point elevation, due to the load delivered to the soil in shear.
4. The value of friction between the pile and the soil is greater than the shear strength of the soil.
5. The shear stress is uniform along the length of pile embedded in the bearing strata.

6. A vertical load applied to the soil produces a maximum vertical pressure on a horizontal plane below it equal to that which would occur if the load spread uniformly over an area within planes through the edges of the loaded area and inclined at 60^0 to the horizontal; except that the load is not assumed to spread laterally beyond the intersection of such 60^0 planes with those from other loads.
7. Each pile in a group supporting a common symmetrically loaded footing, carries the same load.

The first assumption is on the safe side except in the case of unconsolidated fills which would result in added load rather than support. It is intended that the group of strata selected shall be that resulting in the shortest pile.

The second assumption is made because failure in the weakest stratum in the group will destroy the support of all the higher strata and throw more load on the other supporting strata than their properties can justify.

The third assumption is based on the fact that any confined, overloaded soil tends to yield plastically and throw greater load on the adjacent soil.

The fourth assumption is true if the pile is at all rough or has a larger surface area (due to corrugations and the like) than a cylinder, prism or cone immediately outside of it.

The fifth assumption is common. Certainly the shear stress will not vary as the shear strength. It seems, therefore, that the assumption is the best approximation that can be made at this time.

The sixth assumption is a common one. It is easily applied and is felt to give results of accuracy comparable to the rest of the procedure.

The seventh assumption is common design practice. To change it would be an unwarranted refinement.

These simplifying assumptions are analogous to those adopted in other fields of structural design such as that of linear variation of stress in a concrete beam; or that of hinged action at the joints of a riveted steel truss.

Notation

a = area of pile point in sq. ft.
A = maximum effective bearing area of soil at the point of an isolated pile in sq. ft.
c = cohesion in kips per sq. ft.
ϕ = angle of internal friction in degrees.
H_1 = depth of soil above the supporting strata that cannot become submerged, in ft.
H_2 = depth of soil above the supporting strata that can become submerged, in ft.
h = distance piles are driven into supporting stratum in feet.
o = perimeter of pile in feet. For H or I shaped piles, use the perimeter of the enclosing rectangle.
p = allowable bearing pressure in kips per sq. ft.
p_m = maximum vertical pressure in kips per sq. ft.
p_v = actual vertical pressure in kips per sq. ft.
P = pile load in kips.
q = unconfined compressive strength in kips per sq. ft.
s = pile spacing in feet.
N = number of blows of a 140 lb. hammer dropping 30 in. required to drive the sampler one foot in the "Standard Penetration Test".

The first condition is satisfied by fulfilling the requirement for a post or column. Such design is not peculiar to piles and consequently will not be covered here.

It is proposed that the second condition be fulfilled with a factor of safety of 2.

Isolated Piles in Cohesive Soils

The full value of internal friction is not available in cohesive soils until consolidation under the new load is complete. It is therefore omitted from consideration. To fulfill condition 2, then, piles should be driven into the supporting strata a distance determined by the following formula:

$$h = \frac{2(P - pa)}{c\ o} \tag{1}$$

Isolated Piles in Non-cohesive Soils

Non-cohesive soils are dependent upon internal friction and surcharge for shear strength. Since the soil in the immediate vicinity of the pile will have been displaced laterally by driving the pile and more soil must be displaced if movement relative to the soil is to continue, it is proposed that the pressure against the pile be taken as the passive pressure.

Assuming the soil to weigh 0.1 kip per cubic foot and its specific gravity to be 2.70, the shear value becomes $(0.1H_1 + 0.063H_2)\ [\tan^2 (45^\circ + \frac{\phi}{2})\ \tan \phi]$.

To fulfill condition 2 then, piles should be driven into non-cohesive supporting strata a distance determined by the following formula:

$$h = \frac{2\,(P - pa)}{o\ (0.1H_1 + 0.063H_2)\ \ [\tan^2 (45^\circ + \frac{\phi}{2})\ \tan \phi]} \tag{2}$$

If h, as determined by equation 2, is greater than $(1.58H_1 + H_2)$, it will be found that a shorter pile can be obtained by using part of the good soil as if it were overburden rather than support.

The third condition requires that the entire pile load be distributed to the soil at the pile point without exceeding the allowable bearing pressure on the area available.

Consider the cylindrical isolated pile shown in Figure 1 (a). The load transferred from the pile to the soil at each ring, dy, will be:

$$\frac{P - p_m\ \ r}{h}\ dy$$

These loads will produce vertical pressures on plane B-B at the elevation of the pile point. There will also be vertical pressures on the pile point produced by direct bearing. A pressure distribution on plane B-B is shown in section on Figure 1 (b). It can be considered to be made up of two parts, the cylindrical figure, p_m high and having a radius of r directly below the pile point (representing point bearing) and a surrounding cone like solid p_m high at its highest ring (due to loads transferred to the soil by friction). The volume of this solid figure $\int p_v\ dA$ is obviously equal to P.

It is proposed, for convenience, to replace this pressure diagram with one having the shape of a square prism, and of the same volume and altitude, as indicated in section Figure 1 (b) by dotted lines. The height is p_m and the

side of the prism is $\sqrt{\frac{P}{p_m}}$. This distribution is a convenient assumption, since it represents a pile spacing at which the overlapping pressures from adjacent piles would not total more than p_m. The area $\frac{P}{p_m}$ will be called "the maximum effective area of distribution" for an isolated pile.

In accordance with our fifth assumption, the maximum vertical unit pressure at the pile point from the load transferred in friction in one ring is:

$$dp_m = \frac{P - p_m \pi r^2}{h} \quad dy \quad \frac{1}{\pi\left[(.577y + r)^2 - r^2\right]}$$

and

$$p_m = \frac{P - p_m \pi r^2}{h} \int_o^h \frac{dy}{(.577y + r^2) - r^2}$$

Solution of the above equation gives an infinite stress at the periphery of the point. Since such a condition is impossible, the following approximation is proposed, which will give definite results:

Use $\pi(.577y + r)^2$ as the area to which each element of load is delivered in lieu of $\pi\left[(.577y + r)^2 - r^2\right]$. This substitute area includes the point area of the pile instead of excluding it. To compensate for this approximation, all of the load is treated as though it were delivered in shear so that $P - p_m \pi r^2$ becomes P. The new equation is:

$$dp_m = \frac{P}{h} \, dy \, \frac{1}{(.577y + r)^2 \pi}$$

and integrating from o to h:

$$p_m = \frac{P}{3.14r^2 + 1.811 \, r \, h} \tag{3}$$

It will be seen that the denominator of the right hand member in the above formula is the "maximum effective area of distribution" for the loads delivered by an isolated pile. It depends only upon the radius of the pile point, r, and the depth of embedment in the bearing stratum. When $p_m = p$ equation (3) may be rewritten

$$\frac{P}{p} = A = 3.14\, r^2 + 1.811 \, r \, h \tag{4}$$

The required depth "h" to develop a given "effective bearing area", A, can be written

$$h = \frac{A - 3.14\, r^2}{1.811r} \tag{5}$$

When the stratum at the pile point is underlaid by a weaker stratum, the load on the larger available area of the lower stratum must be checked as shown in Figure 2 to insure that the fourth condition is complied with.

Groups of Piles

The requirements to satisfy the 1st and 2nd conditions are no different for piles in groups than they are for isolated piles.

In a group of piles, the effective bearing area of a pile cannot be greater than A as given by equation (4); in addition, the effective bearing area for an interior pile cannot be much greater than that enclosed in a line midway between the piles.

For simplicity a square spacing will be assumed in what follows. The second area mentioned above then becomes s^2. See figure 3.

It is apparent that if a value of s^2 is used such that

$$s^2 \geq \frac{P}{p} \tag{6}$$

the third condition will be satisfied for a group of piles by taking the value of the group as the value of one pile multiplied by the number of piles in the group. It is likewise apparent that if $s^2 < \frac{P}{p}$; the allowable load per pile in groups must be decreased in accordance with formula 7.

$$P \leq ps^2 \tag{7}$$

There remains the necessity of satisfying the 4th condition. This can be accomplished in the following manner.

If any stratum below the point has a "p" smaller than that at the points, it should be checked to see that the actual pressure on it does not exceed its value. In case it is found that the lower stratum will be overloaded, the group of piles should be respaced to spread their load over a sufficiently larger area. (See Figure 3.)

Evaluation of Soil Properties

It is obvious that to make use of the formulas developed in the preceding chapter certain properties of the soil must be evaluated in each case. It serves no purpose to say that some of these properties cannot be evaluated with sufficient assurance. The fact is that any design is uncertain to the extent that these properties are uncertain.

The measurable properties of the soil which are required are the unconfined compressive strength of cohesive soils; the relative density and angle of internal friction for non-cohesive soils. From them will be derived the allowable bearing value and the shear value of the soils, which will be applied directly.

The above properties can be obtained from undisturbed samples. They can also be estimated from the results of a method of soil exploration called the "standard penetration test". The procedure will be outlined in terms of that test. In cases of doubt, the penetration test should be supplemented by tests on undisturbed samples. However, the process of using the values obtained in that way instead of those estimated from the standard penetration test will be obvious.

A great deal of what follows is based upon information contained in "Soil Mechanics and Engineering Practice" by Terzaghi and Peck. Reference will be made to that work throughout this part of the treatise and for the sake of brevity it will hereinafter be called "Terzaghi and Peck".

The Standard Penetration Test is described on page 265 of that volume. Table 23 on page 430 of "Terzaghi and Peck" gives average unconfined compressive strength of clay in terms of N, the number of blows required to drive the sampler one foot in the standard penetration test. If the values be translated into kips per sq ft, it will be found that a straight line having the

equation, $q = .267N$ fits them very well. It is proposed therefore that $.267N$ be taken as the value of the unconfined compressive strength of cohesive soil.

"Terzaghi and Peck" states that the allowable bearing value of a clay with a factor of safety of 3 against breaking into the soil is $1.2\ q$ for square footings and $0.9\ q$ for continuous footings. This would be $.32N$ for square footings and $.24N$ for continuous footings. It is proposed then, that "p" in cohesiv soils be taken as $\frac{N}{4}$ under continuous pile groups and as $\frac{N}{3}$ under square pile groups. It is further proposed that "p" for rectangular pile groups of width B and length L be taken as $\frac{N}{4}(1 + 0.3\frac{B}{L})$. When the loads from several groups merge at a plane below the pile points as in figure 3, the shape of the lower area should be used in establishing the allowable pressure.

It has been proposed to use only the cohesion of cohesive soils as their shear strengths. Because of the slow consolidation of such soils they may be fully loaded before any appreciable internal friction can develop. The shear strength then will be taken as one-half the unconfined compressive strength or $.133N$.

Figure 177 on page 423 of "Terzaghi and Peck" gives allowable bearing values on sand for various widths of footing on the basis of the standard penetration test. The bearing values decrease as the width of the footings increases. The decrease for footings wider than 10 feet is relatively small. It is proposed that on non-cohesive soils, at depths which require piles, the allowable bearing value be taken as $\frac{N}{5}$ kips per sq ft except that on fine sands or silts that may become submerged it be taken as $(1.5 + \frac{N}{10})$ kips per square foot, but not more than $\frac{N}{5}$ (see pages 425 and 426 of "Terzaghi and Peck").

The shear value for non-cohesive soils will be the normal pressure at the face of the pile multiplied by the tangent of the angle of internal friction. Table 10 on page 294 of "Terzaghi and Peck" gives values of relative density for sand in terms of N.

Table 7 on page 82 of the same work gives representative values of ϕ, the angle of internal friction, in terms of density, shape of grain and grading. The values are 28.5° for loose sand and 35° for dense sand when the grains are rounded and uniform in size; and 34° for loose sand and 46° for dense sand when the grains are angular and well graded.

Take $N = 7$ as an average value for loose sand and $N = 40$ as an average value for dense sand. The formula $\phi = \sqrt{12N} + 15$ gives $\phi = 24.2^\circ$ for loose sand and $\phi = 37^\circ$ for dense sand. The formula $\phi = \sqrt{12N} + 25$ gives 34.2° for loose sand and 47° for dense sand. It is proposed therefore that $\sqrt{12N} + 15$ be used as the value of ϕ for sand with rounded grains of uniform size; and that $\sqrt{12N} + 25$ be used for sand with angular grains well graded. It is further proposed that for sand with rounded grains, well graded or angular grain: of uniform size the value of ϕ be taken as $\sqrt{12N} + 20$.

The dotted lines in figure 4 show the values of $\tan\phi(45 + \frac{\phi}{2})\tan\phi$ for the three gradings of sand which we have assumed, where $\phi = \sqrt{12N} + 15$, $\phi = \sqrt{12N} + 20$, $\phi = \sqrt{12N} + 25$. The straight lines which approximate the curves very closely, correspond to the linear equations noted beside them. It appears justifiable therefore to replace $\tan\phi(45 + \frac{\phi}{2})\tan\phi$ with the right hand member of the corresponding equation within the range $0 < N < 60$.

Design Procedure

Equations 1 and 2 can now be made useable by the substitution therein of soil properties in terms of N and a design procedure for a specific case can be based thereon. If the soil properties are measured by other means, they can be applied directly or an artificial N can be derived by means of the relations developed in the previous chapter. In what follows a soil investigation adequate to define the soil properties under the proposed building is assumed. The following procedure is proposed.

1. Evaluate the strata of the soil that will be used to support the piles and all strata below as follows:

   For cohesive soils (clay and plastic silt)

   $$p = \frac{N}{4} \tag{8}$$

   For non-cohesive soils except submerged fine sand or silt

   $$p = \frac{N}{5} \tag{9}$$

   For submerged fine sand or silt

   $$p = 1.5 + \frac{N}{10} \text{ but not over } \frac{N}{5} \tag{10}$$

2. Compute the penetration necessary to transfer the pile load to the soil as follows:

   a. In cohesive soils - see Figure 1.

   $$h = \frac{P - 0.25Na}{0.0665No} \tag{11}$$

   b. In non-cohesive soils

   $$h = \frac{P - 0.20Na}{0.0315\ K\ (1.58H_1 + H_2)\ o} \tag{12}$$

Note: As previously stated if the value of "h" determined as above exceeds $(1.58H_1 + H_2)$ a shorter pile can be found by using a part of the depth of the good soil as overburden rather than as supporting soil. In this case, use $(1.58H_1 + H_2)$ not less than $\frac{P - 0.20Na}{0.0315Ko}$.

It will be found that this is also the value of h so that if h as determined by formula (12) is less than $(1.58H_1 + H_2)$, the total length of the shortest permissible pile is $2\ \frac{P - 0.20Na}{0.0315Ko} - .58H$. In both (11) and (12)

K = a constant depending upon N, the grain shape and the grading of a non-cohesive soil.

$$K = \begin{cases} (0.46 + 0.067N) & \text{for round grains of uniform size.} \\ (0.74 + 0.098N) & \text{for round grains, well graded or sharp grains of uniform size.} \\ (1.15 + 0.147N) & \text{for sharp grains, well graded.} \end{cases}$$

3. Check "h" to see that it is great enough so that entire pile load will be spread over sufficient area at the pile point so that "p" for the soil below the pile points is not excessive.

   $$h = \frac{\frac{P}{p} - a}{1.811\ r} \tag{13}$$

   For square or octagonal piles use the radius of a circle having the same area as the pile.

4. Compute the minimum spacing of piles in groups

$$s = \sqrt{\frac{P}{p}} \qquad (14)$$

5. If a lower stratum is weaker than the stratum carrying the pile points, investigate the load upon it to see that the pressure is not excessive. This may be done on the assumption that the total load of the pile group is applied uniformly at the elevation of the pile points to an area equal to that of the pile cap, and by assuming that this load will spread uniformly on any lower plane within a figure bounded in that plane by planes extending from the edge of the above area at angles of 60° with the horizontal. The area considered as supporting the load shall not extend beyond the intersection of such 60° planes from adjacent pile groups. See figure 3.

The above procedure will give for each boring a point elevation and the elevation at which the pile point enters a material capable of carrying a whole pile load on an area of s^2. Obtain these two values for each pile group by interpolation. Call the first, "contract point elevation" and the second, "highest permissible point elevation". These two values should be given on the drawings for each pile group.

Examples

The following examples are intended to illustrate the design procedure. The design will be made for the three illustrative conditions shown in Figure 5.

Suppose a foundation is to be supported on pile groups consisting of nine 30 ton, 10" diameter piles and that the groups are spaced 20 feet on center in two directions at right angles to each other. Determine the length and spacing of piles if each boring in turn is considered representative of the whole site.

Boring A

The p's are given on the boring. The soil is cohesive.

$o = 0.833 \times 3.14 = 2.62$ ft; $a = 0.833^2 \times .785 = 0.543$ sq ft

Try using the soil below 17 feet for support.

N = 22

$$h = \frac{60 - 0.25 \times 22 \times .543}{0.0665 \times 22 \times 2.62} = 14.9 \text{ ft} \qquad (11)$$

Total length = 14.9 + 17 = 31.9 ft.

Try using the soil below 27 feet for support.

N = 30

$$h = \frac{60 - 0.25 \times 30 \times .543}{0.0665 \times 30 \times 2.62} = 10.7 \text{ ft} \qquad (11)$$

Total length - 27 + 10.7 = 37.7 ft

Use 31.9 foot piles. (This is contract length.)

The pile point will be in material having a p of 7.5 ksf.

Using formula (13) -

$$h = \frac{\frac{60}{7.5} - .543}{1.811 \times 0.417} = 9.92 \text{ ft.}$$

This is the depth the pile must be driven into the supporting material to spread its load over the material at its point. We have a penetration of 14.9 ft which is satisfactory.

The minimum allowable spacing of piles is $s = \frac{60}{7.5} = 2.83$

Use 31.9 foot piles; 3'-0" o.c.
Give contract point elevation 31.9 ft deep.
Give highest permissible point elevation 27.0 ft deep.

The points of the nine pile group will be 18' above the very stiff clay capable of carrying 6.25 kips per sq ft. The nine pile footing will be about 9 feet on a side. Consider the 270 tons or 540 kips applied to a 9' square at the pile points. This load can spread to a 20-foot square (see Figure 3) which will occur 9.5 feet below the pile points. The intensity of load on the 20-foot square will be $\frac{540}{400} = 1.35$ kips per square foot which is satisfactory for the 25-blow soil.

Boring B

Again; $o = 2.62$ ft; $a = 0.543$ sq ft.
The soil is non-cohesive. Assume that an examination of the samples shows round grains, well graded so that $K = (0.74 + 0.098N)$. $H_1 = 10'$.
Try $H_2 = 5'$; $N = 25$

$$h = \frac{60 - 0.20 \times 25 \times .543}{0.0315\ (0.74 + 0.098 \times 25)\ (1.58 \times 10 + 5)\ 2.62} = 10.5 \text{ ft.} \qquad (12)$$

The total length is 10.5 + 5 + 10 = 25.5 ft. to transmit the load to the soil.

Applying formula (13), a penetration of $h = \dfrac{\frac{60}{5} - 0.543}{1.811 \times 0.417} = 15.2$ ft.

is required to spread the load on 25-blow soil. Use a pile reaching the dense sand and gravel N = 40.

A penetration of 12 feet in 25-blow material will give an effective area at the top of the 40-blow soil of 9 sq ft (limited by pile spacing), capable of carrying 5 kips per sq ft or 45 kips. The pile should be driven far enough into the 40-blow material to spread the remaining 15 kips. The material is already loaded with 5 Ksf so the additional length will be

$$\frac{\frac{15}{(8-3)} - .543}{1.811 \times 0.417} = 5.9 \text{ ft.}$$

The minimum spacing is $s = \sqrt{\frac{60}{8}} = 2.74'$.
Use piles 33 feet long; 3'-0" o.c.
The lower strata are o.k. by inspection since for

$p = 7$; $s = \frac{60}{7} = 2.93$ ft.

Give contract point elevation 33 feet deep.
Give highest permissible point elevation 27 feet deep.

Boring C

Again; $o = 2.62$ ft; $a = 0.543$ sq ft.
Assume the dense sand to be sharp and well graded so that $K = (1.15 + 0.147N)$ $H_1 = 10$
Try $H_2 = 0$; $N = 6$

$$h = \frac{60 - 0.20 \times 6 \times 0.543}{0.0315\ (1.15 + 0.147 \times 6)\ (1.58 \times 10)\ 2.62} = 22.4 \text{ ft.}$$

$$22.4 + 10 = 32.4 \text{ ft.}$$

h is greater than $(1.58\ H_1 + H_2)$

The shortest pile for N = 6 will be -

$$2\sqrt{\frac{60 - 0.20 \times 36 \times 0.543}{0.0315\ (1.15 + 0.147 \times 6)\ 2.62}} - 5.8 = 31.8 \text{ ft.}$$

Try H_2 = 10; N = 35

$$h = \frac{60 - 0.20 \times 35 \times 0.543}{0.0315\ (1.15 + 0.147 \times 35)\ 25.8 \times 2.62} = 4.2 \text{ ft.}$$

Total length = 10 + 10 + 4.2 = 24.2 ft.

Piles 24.2 ft long will penetrate the dense sand 4.2. To spread the load on material having a p = 7 however, the penetration should be,

$$h = \frac{\frac{60}{7} - 0.543}{1.811 \times 0.417} = 10.6'.$$

$$s = \sqrt{\frac{60}{7}} = 2.93 \text{ ft.}$$

Try piles 30.6' long, 3'-0" o.c.

A nine-pile footing will deliver 540 kips to a 9 ft square, 10.6 feet below the top of the dense sand. At the top of the plastic silt, 4.4 feet lower, there will be a load of 540 kips on a 14.1 foot square. The pressure there will be $\frac{540}{199}$ = 2.7 kips per sq ft. This material is good for 1.5 kips per sq ft. To reduce the load to 1.5 ksf we must spread it over $\frac{540}{1.5}$ = 360 sq ft or a 19 foot square. Working backward, the load of the 9 piles should be spread over a [19 - (2 x 4.4 x 0.577)] = 13.9 ft square. $\frac{13.9}{3}$ = 4.63 feet.

Use piles 30.6 feet long, 4'-8" o.c.
Give contract point elevation 30.6 feet deep.
Give highest permissible point elevation 20.0 feet deep.

So far the treatment has concerned itself with cohesive soils and non-cohesive soils. Most soils encountered in nature are mixed. It has been the writer's practice to treat each soil in accordance with its predominant characteristics. This is believed to be on the safe side because the mixture adds to the combination an element of strength that is not considered in the formula used. Cohesive materials in predominantly granular soils add a cohesion not assumed in formula 12. Likewise the presence of grit in a clay adds an internal friction that is not used in formula 11. In cases of doubt the pile length can be computed both ways and an intermediate value selected by inspection. Ordinarily the two lengths will not be greatly different.

Field Procedure

The contract point elevations for each pile group, obtained by interpolation between borings, are the elevations that should be expected to result in adequate piles if the soil strata varied linearly between borings. This would fix the estimated pile lengths. However it is obvious that, actually, due to irregular variation of strata between borings, some piles probably should be driven deeper and others probably could safely be stopped at higher elevations.

Since it is impracticable to have a boring at each pile group, it is necessary to set up some criterion to be used in the field to determine when each pile has been driven far enough. Here, and here only, it is proposed to use driving resistance as a criterion. Note carefully the limitations.

At each of certain borings which are selected to bracket the foundation and represent the range of conditions on the site, have the contractor drive a control pile or pile casing of the type to be incorporated in the foundation with the same driving equipment that will be used for the rest of the project to the contract point elevation computed for that boring and note the number of blows required for the last foot of driving. The number of control piles required will depend upon the size of the project but there should preferably be at least three.

If any of the control piles drive so easily that there is doubt of its load carrying capacity, a load test should be applied to it. This has not happened within the writer's experience.

Otherwise, or if the test has proved the control piles satisfactory, average the blows required for the final foot on the control piles and require all piles to be driven to that resistance.

There remains the possibility that a crust may cause some piles to reach the required resistance while there is still soft material below. To avoid this, and to insure as well as possible that all soil below the pile points is satisfactory for groups of piles, require all piles to be driven to the highest permissible point elevation if that is lower than the point elevation at which the specified resistance is encountered.

As a basis of equitable contract settlement:

1. The lump sum bid should be on the basis of the aggregate length of piles to reach the contract point elevation.
2. There should be a price per foot specified for aggregate overrun and another price per foot for aggregate underrun.

Possible Refinements

The design procedure as outlined and illustrated has admittedly forgone certain advantages in order to make it compact and easy to apply. It would be justifiable under some circumstances to take advantage of some or all of the possible refinements.

The instructions call for evaluating "p" for cohesive soils as $\frac{N}{4}$. This corresponds to the value in the reference for a strip footing. Where the loaded area is square, "p" can safely be taken as $\frac{N}{3}$ and where it is rectangular as $\frac{N}{4}(1 + 0.3\frac{B}{L})$ (where B is the width and L is the length). In this connection it must be remembered that the shape of the loaded area often is different at one depth than at another. For a row of square pile groups for instance, the area will be square and "p" may be taken as $\frac{N}{3}$ at the pile points. Further down where the loads from the groups overlap the supporting area will be continuous or rectangular and "p" should be taken as $\frac{N}{4}$ or $\frac{N}{4}$ $(1 + 0.30\frac{B}{L})$.

In formulas 11 and 12, "N" appears both in the numerator and the denominator. The N in the denominator determines the shear strength of soil and should therefore be the least N in the strata used for support. The N in the numerator determines the bearing value on the point of the pile and could be taken as the N of the stratum at the pile point. This refinement will be of importance when the point is in much harder material than the rest of the supporting layers.

In Formulas 11 and 12, the perimeter of the pile is used and in formula 13 the radius appears. This practice requires no modification for cylindrical or prismatic piles.

For tapered piles it will obviously be on the safe side to use the "o" and "r" of the point of the pile. In case of deep penetration into the supporting layers it is proposed that the average dimensions within the supporting layers or the dimensions within the weakest supporting layer, whichever are less, be used. This will introduce a cut and try procedure, since the dimension to be used to find "h" depends upon "h". In such a case a value of "o" and "r" would be assumed from inspection and then compared with the critical values for the resulting penetration.

The maximum assumed spread for a pile in a group has been held to an area of s^2 in the procedure while it has been assumed that a single pile will spread its load over an effective area of $(3.14^2 + 1.811\ rh)$ square feet. The first assumption is on the safe side and nearly true for large pile groups. Obviously however, it is not nearly as good for small groups. Consider for instance a two pile group with $r = .417$, $s = 3$ and $h = 20$. The procedure permits us to spread the pile load over 9 square feet. It indicates an area of spread for a single pile of $(3.14 \times 0.417 + 1.811 \times 0.417 \times 20 = 15.6$ sq ft. (4)

It seems reasonable to interpret the 15.6 sq feet as meaning that we can consider the load of the group spread on a rectangular area the sides of which are $\sqrt{15.6} = 3.95$ feet from the centers of the piles. This would permit us to make the area of spread for the two piles $3.95 \times (3 + 3.95) = 27.5$ square feet instead of 18 sq ft. In this case, the shear on the periphery of the group should be checked.

The method lends itself to the construction of design charts for the more commonly used pile sizes and capacities. Figures 6, 7 and 8 illustrate such charts. Several sizes of pile can be grouped on one chart such as 6 or 8. A separate chart such as 7 is needed for each size of pile and each value of E.

CONCLUSION

The value of the foregoing method can only be proved by tests including tests of groups of piles and by experience.

Load tests in sufficient detail to permit a careful check of the method are scarce and expensive. As they become available, the method should be checked against them. Any tests made should be reported in sufficient detail to permit comparison.

Experience with the method to date, including a total of 6518 piles varying in length from 10 to 50 feet in both cohesive and non-cohesive soils indicates:

1. Satisfactory load carrying capacity.
2. Reasonable pile lengths compared to those carrying comparable loads under other buildings in the vicinity.
3. Excellent correlation between estimated and actual aggregate length of piling.

In closing, the author wishes to emphasize that the method outlined must be used <u>with</u> engineering judgment - not in lieu of it.

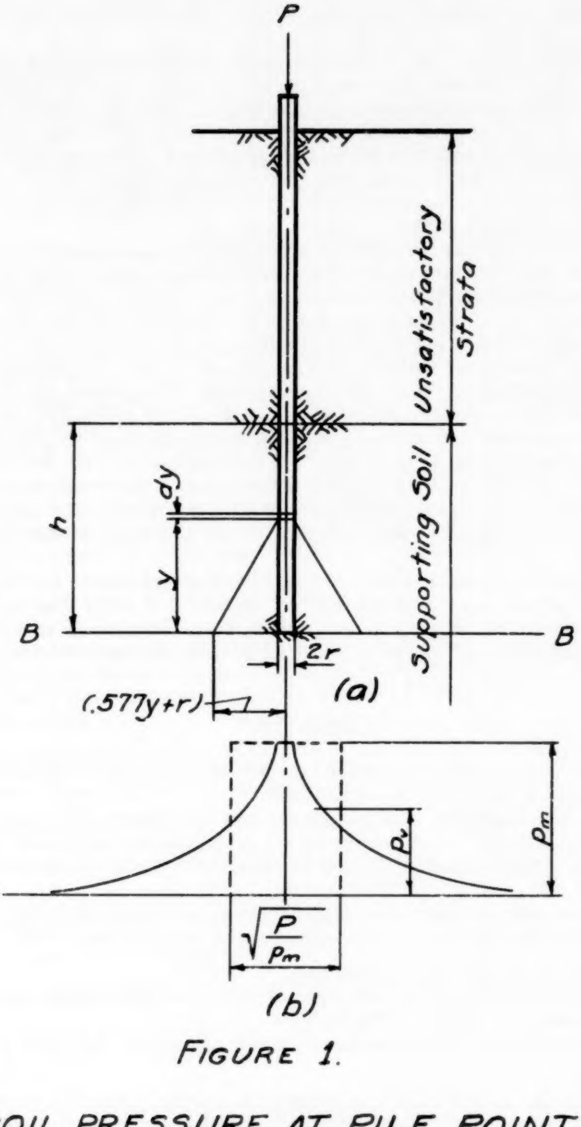

FIGURE 1.

SOIL PRESSURE AT PILE POINT

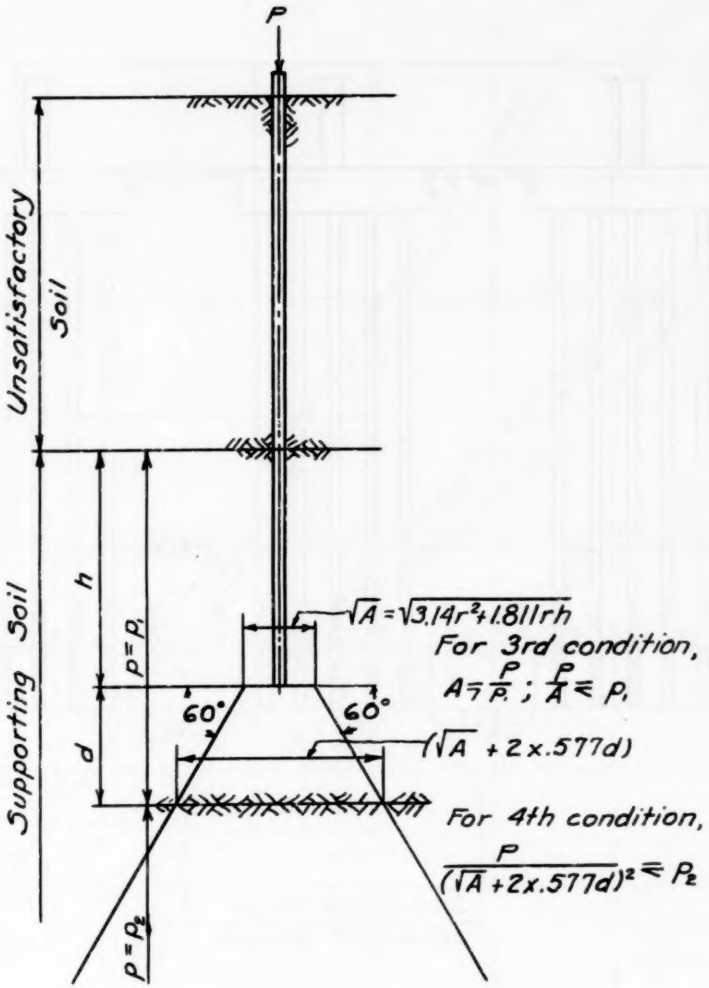

FIGURE 2.
LOAD DISTRIBUTION BELOW ISOLATED PILE

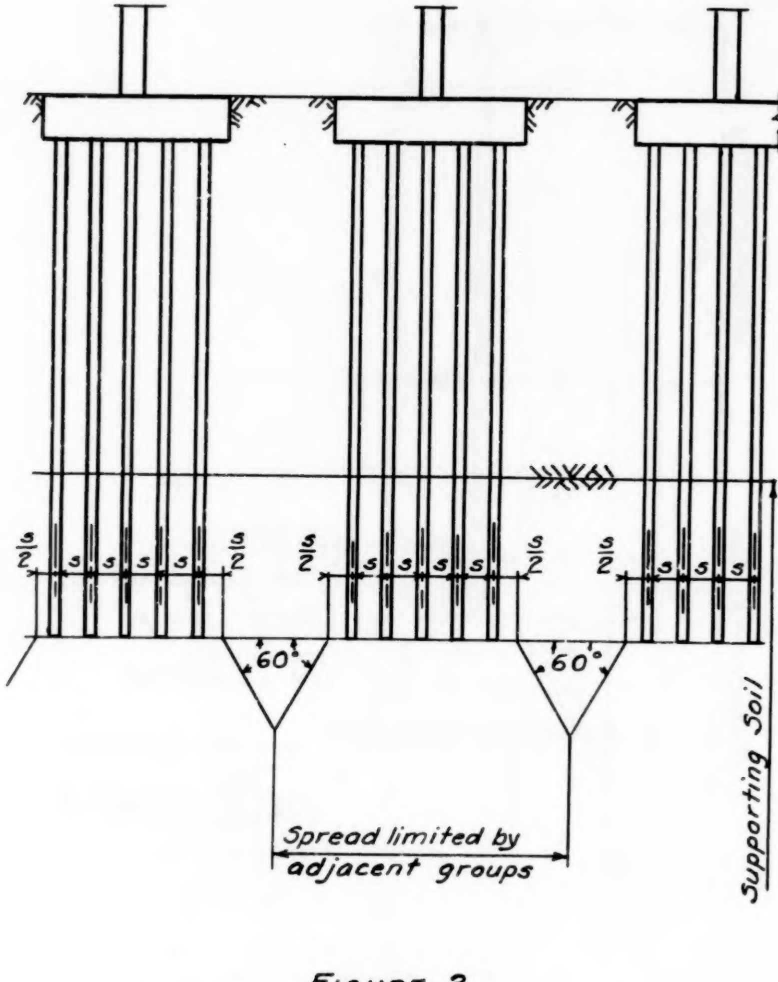

FIGURE 3.
LOAD DISTRIBUTION BELOW PILE GROUPS WHEN $s \leqq \sqrt{\frac{P}{p}}$

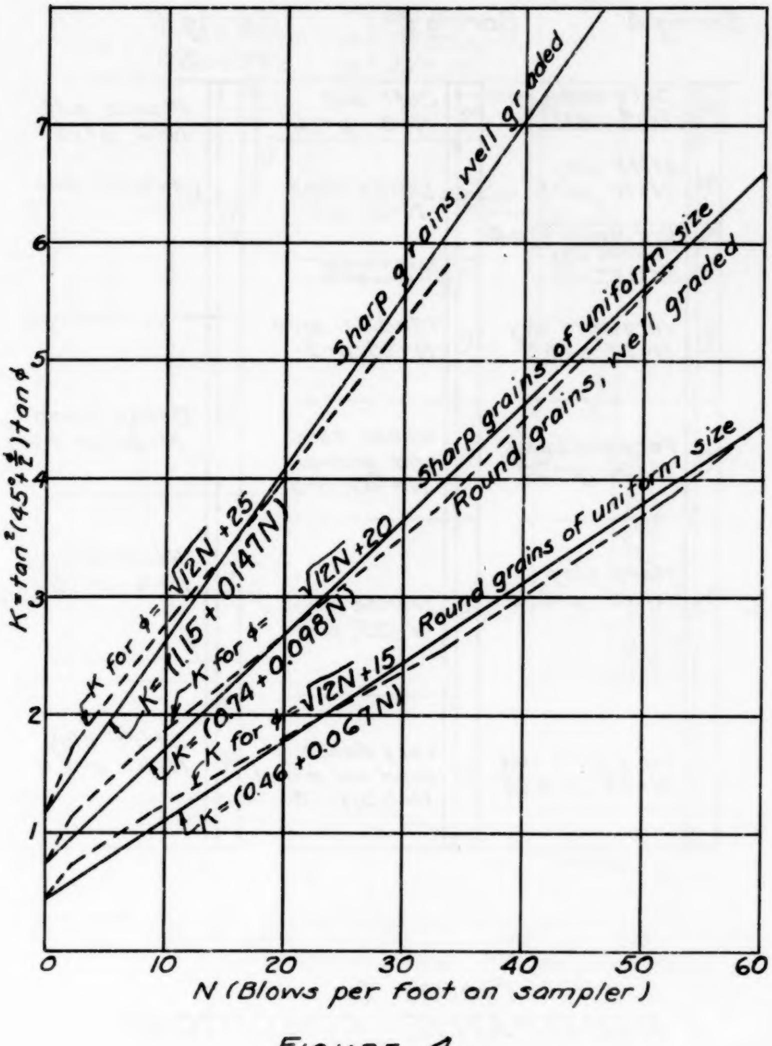

FIGURE 4.
K FOR NON-COHESIVE SOIL

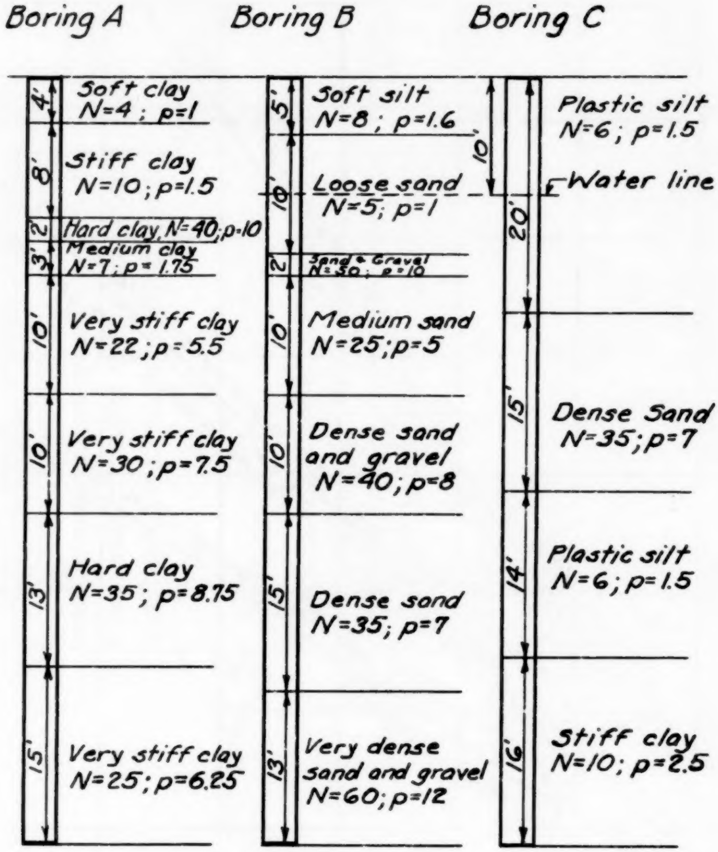

FIGURE 5.
ILLUSTRATIVE CONDITIONS

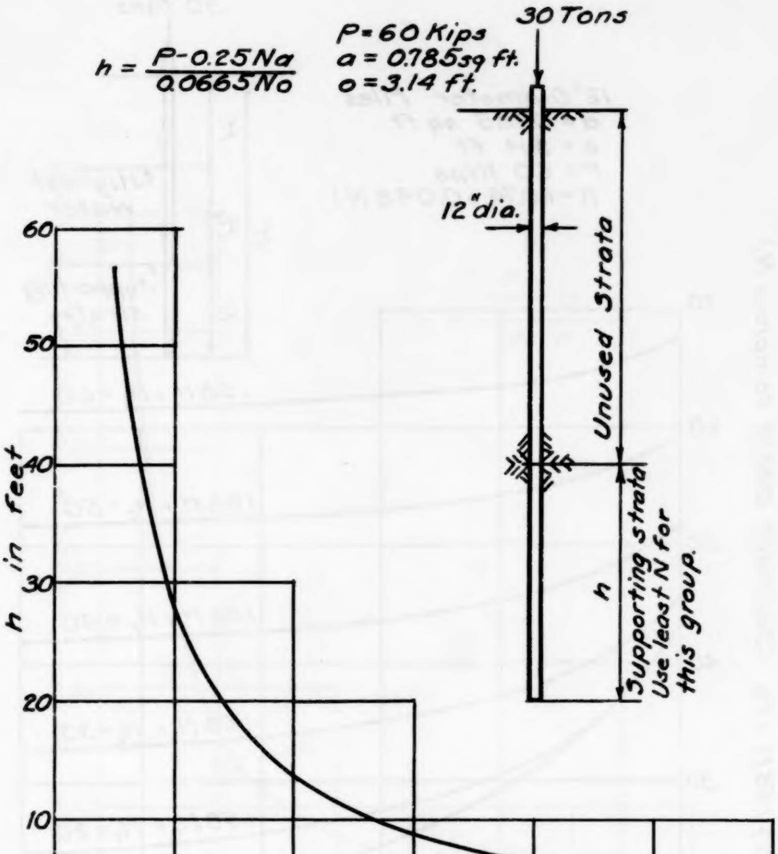

FIGURE 6.
30-TON PILES IN COHESIVE SOIL

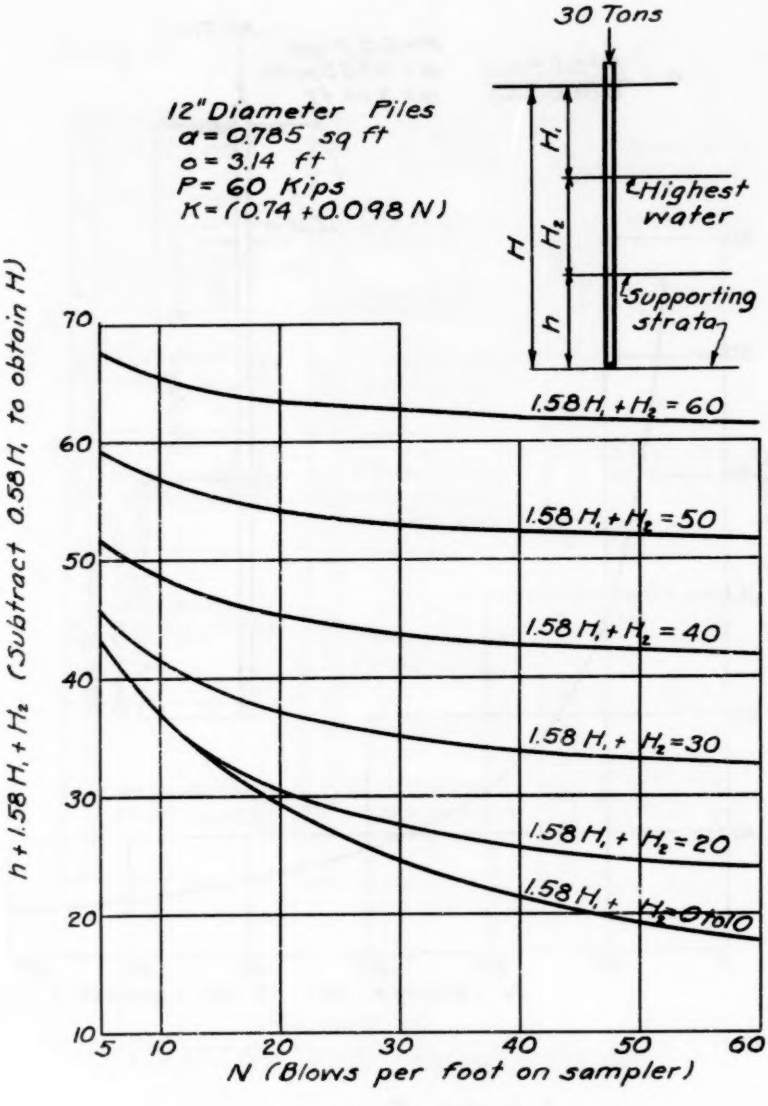

FIGURE 7.
30-TON PILES IN NON-COHESIVE SOIL

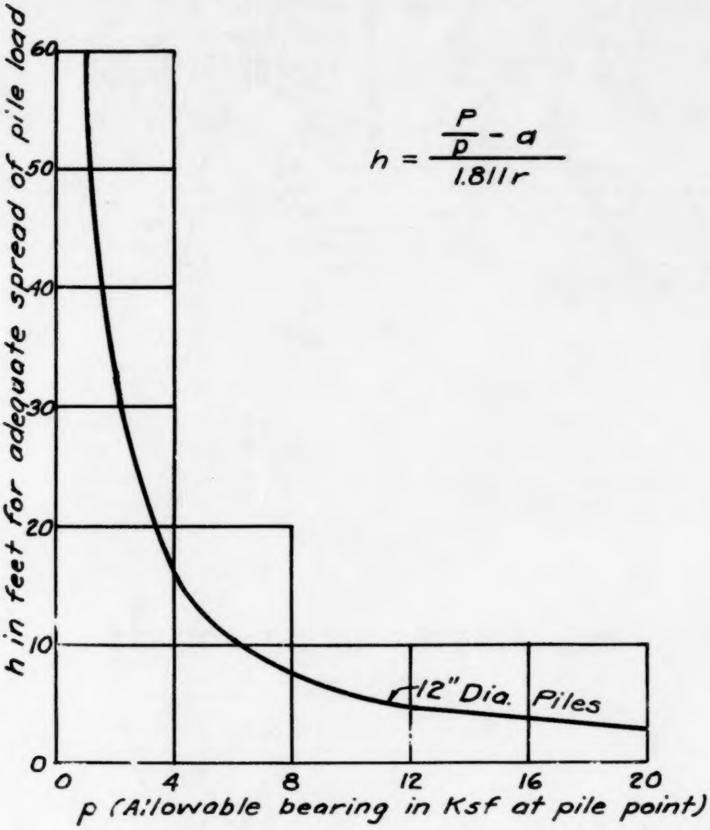

FIGURE 8.
MINIMUM h FOR LOAD SPREAD
30-TON PILES